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Soutsos, M., Hatzitheodorou, A., Kanavaris, F., & Kwasny, J. (2019). Compressive strength estimates of adiabatically cured concretes using maturity methods. *ASCE Journal of Materials in Civil Engineering*, 31(7). [https://doi.org/10.1061/\(ASCE\)MT.1943-5533.0002757](https://doi.org/10.1061/(ASCE)MT.1943-5533.0002757)

Published in:
ASCE Journal of Materials in Civil Engineering

Document Version:
Peer reviewed version

Queen's University Belfast - Research Portal:
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Compressive strength estimates of adiabatically cured concretes using maturity methods

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Abstract

The strength development of standard and adiabatically cured concretes was determined. The concrete mixes were of 28-day cube strengths of 50 and 30 MPa and also had Portland cement (PC) replaced partially with fly ash (FA) and ground granulated blast-furnace slag (GGBS) at 30% and 50%, respectively. The peak adiabatic temperature was effectively reduced with GGBS addition but was only reduced with FA addition for the lower w/b concrete. Considerable early age strength enhancements resulted from the adiabatic curing regime. The Nurse-Saul and Arrhenius based maturity functions were used to estimate the increases in early age adiabatic strength. The Nurse-Saul function underestimated the

effect of high early age curing temperature for all concretes but to a greater extent for those with GGBS and FA whilst the Arrhenius based, which allows for the consideration of an “apparent” activation energy, gave more accurate estimates. Strength estimates for adiabatically cured concretes and isothermally (50 °C) cured mortars were also compared indicating that the latter might have been affected by the detrimental effect of high curing temperatures starting from early age.

Keywords

“Apparent” activation energy, Maturity functions, Compressive strength development and estimates, Fly ash, Ground granulated blast-furnace slag.

Introduction

There is a need to understand and quantify the effects of temperature on the early age strength development of concrete mixtures. This need, which has been recognised for a long time from researchers and engineers, has been mainly associated with: a) determining the elevation of curing temperature necessary to achieve the required early age strength (Saul, 1951) which will enable safe lifting of precast concrete structural elements as early as sixteen to eighteen hours after casting and, b) predicting the real-time strength on-site, particularly during cold weather concreting, to allow safe formwork striking and removal of props and ultimately, avoid collapses like the Willow Island one (1978) which resulted in 51 deaths (Lew et al. 1979; Feld and Carper, 1997). This can be accomplished with maturity functions which aim to account for the combined effect of temperature and time on the strength development of concrete (Barnett et al. 2007a; Brooks et al. 2007; Galobardes et al. 2015, Sofi et al. 2012, Yikici et al. 2015; Soutsos et al. 2016a).

Saul (Saul, 1951) proposed a single factor, *i.e.* “maturity”, to be indicative of the concrete strength irrespective of the combination of time and temperature that make up that maturity:

$$M = \sum_t (T - T_0) \cdot \Delta t \quad \text{Equation 1}$$

where: M is the maturity (°C·days),
T is the average temperature (20 °C for standard curing) over the time interval Δt (°C),
T₀ is the datum temperature (°C),
 Δt is the time interval (days).

Equation 1, which is what has become known as the Nurse-Saul maturity function, assumes that the strength development rate varies linearly with curing temperature. It can be also be expressed in a form of an equivalent age, in which a given curing history (reckoned in temperature-time) corresponds to an equivalent age of curing at a reference temperature, as given by Equation 2.

$$t_e = \frac{\sum (T - T_0)}{(T_r - T_0)} \cdot \Delta t \quad \text{Equation 2}$$

where: t_e is the equivalent age at the reference temperature (days),
T_r is the reference temperature (°C).

The concept of equivalent age, which was originally introduced by Rastrup (1954), has become particularly convenient when it comes to using other formulations besides Equation 1 to account for the combined effects of temperature and time on the strength development of concrete.

The assumption that the strength development rate follows the Arrhenius principle leads to the maturity function shown in Equation 3, which is referred to as Arrhenius function in this study (Freiesleben and Pedersen, 1977).

$$t_e = \sum e^{-\frac{E_a}{R} \left(\frac{1}{T_a} - \frac{1}{T_s} \right)} \cdot \Delta t \quad \text{Equation 3}$$

where: t_e is the equivalent age (days),

T_a is the average temperature of concrete during time interval Δt (K),

T_s is the specified reference temperature (K),

E_a is the “apparent” activation energy (J/mol),

R is the universal gas constant (J/K·mol).

The determination of the “apparent” activation energies can be achieved using “equivalent” mortar samples, as recommended by ASTM Standard C1074-98 (ASTM, 2011) and the results can be, subsequently, applied to the concrete under investigation. This requires the determination of strength development under at least three curing temperatures.

Regression analysis is needed in order to relate concrete strength to age or maturity index (Carino, 2004; Freiesleben and Pedersen, 1985; Carino and Tank, 1992). The hyperbolic function proposed by Carino (Carino and Tank, 1992) (Equation 4) is the one suggested by ASTM C1074-11 (ASTM, 2011).

$$S = \frac{S_u \cdot k \cdot (t - t_0)}{1 + k \cdot (t - t_0)} \quad \text{Equation 4}$$

where: S is the compressive strength at age t (MPa),
 S_u is the ultimate compressive strength at temperature T (MPa),
 k is the rate constant (1/days),
 t is the test age (days),
 t_0 is the age at which compressive strength development is assumed to begin (days).

The rate constant, k , the ultimate strength, S_u , and the age at which strength development begins, t_0 , of each mortar mixture is determined at all investigated curing temperatures through regression analysis.

ASTM C1074-11 (ASTM, 2011) recommendation for the calculation of the “apparent” activation energy, E_a , is to plot $\ln(k)$ against $1/T_{abs}$ (given in 1/Kelvin), with T_{abs} being the absolute curing temperature. The slope of the trend line, designated as $-Q$, is then obtained from regression analysis and the “apparent” activation energy (E_a) of the mix under investigation will be equal to $Q \cdot R$, with R being the ideal gas constant equal to 8.31 J/K·mol.

The concrete mixes investigated in the work described herein were those used for the Department of Trade and Industry (DTI) concrete core project (The Concrete Society, 2004). These were of 30 and 50 MPa 28-day compressive strength and included partial Portland cement (PC) replacement with ground granulated blast furnace slag (GGBS) and fly ash (FA) at 50 and 30% cement replacement levels, respectively. The mixes were replicated in the laboratory in order to compare the in-situ strength development, as determined from testing cores obtained from various structural elements, with that of laboratory cast cubes cured in a computer controlled matched curing tank which replicated the in-situ temperature history:

- Phase I investigated the effect of in-situ temperature on the early age strength development of concretes with GGBS and fly ash (Soutsos et al. 2016), and,
- Phase II investigated the effect of isothermal curing temperature on the strength development of mortar mixes with GGBS and FA (Soutsos et al. 2017).

Accurate strength estimates were obtained for in-situ temperatures around 20 °C (during summer and below this down to 10 °C during winter) and peak temperatures of 51 °C and 61 °C are only reached 33 to 60 hours after casting, e.g. in partially insulated large concrete blocks 1.5 x 1.5 x 1.5m cast during summer (Soutsos et al. 2016). The effect of high early curing temperatures of up to 50 °C immediately after casting and cured isothermally at such temperature until tested was investigated in Phase II (Soutsos et al. 2017). The Nurse-Saul function was found to underestimate the early age strength development at higher temperatures whilst the Arrhenius function overestimated them (Soutsos et al. 2017).

There is a need to determine the reasons why maturity functions become inaccurate when high early age curing temperatures are used. Such curing temperatures may nowadays be needed with the new types of cements (CEM II – PC with GGBS, FA, limestone or silica fume, CEM III – PC with GGBS, CEM IV – PC with medium-high volume of siliceous FA and CEM V – composite cement) that have gained popularity due to their lower than CEM I (neat PC) carbon footprint. CEM III/B which contains 66-80% GGBS is also required for exposure class XS1, XS, and XS3 (corrosion induced by chlorides from sea water) (BSI, 2016) and if specified then it is expected to cause production issues for precast concrete factories. The required early age strength as per (BSI, 2014), e.g. 15 and 24 MPa for reinforced and prestressed concretes, respectively, at 16 to 18 hours after casting, may only be achieved with high early age curing temperatures as soon as concrete is cast and without the “delay period” before the “temperature rise period” as is normally recommended for

precasting works (Neville and Brooks, 2010). Thermal activation by use of heated mix water has also been suggested (Reddy and Soutsos, 2016).

Earlier work (Soutsos et al. 2017) indicated that the Arrhenius function overestimates early age strength of concretes cured at elevated temperatures and that the reason for this is the detrimental effect on compressive strength. This was believed to be for later age compressive strengths but it now appears to start from a very early age. The early and later age detrimental effects, reported also elsewhere (Sajedi and Razak, 2011; Lothenback et al. 2007; Brooks et al. 2007; Carino, 2004; Kim et al. 1998), need to be understood and incorporated into maturity functions to improve strength estimates at early ages. The aim of this investigation was therefore to quantify the detrimental effect of high early age curing temperatures on the compressive strength estimates at early ages, particularly those of the Arrhenius function.

Materials and experimental procedures

Materials

The objective was to use cement additions, which are also known as cement replacement materials (CRMs), and aggregates that were as similar as possible to those originally used by DTI project.

Portland cement with a compressive strength of 57 MPa at 28 days (tested according to BS EN 196-1-2005 (BSI, 2005)), was supplied in bags by British Lime Industries. PC conformed to the requirements of BS EN 197-1:2011 (BSI, 2011). Two CRMs were used to partially replace PC, namely GGBS and FA. GGBS, conforming to BS EN 15167-1:2006 (BSI, 2006), was supplied in bags by the Appleby Group. FA, conforming to BS EN 450-1:2005 (BSI, 2012), was supplied in sealed plastic containers by a coal burning power station,

in Warrington, UK (Soutsos et al. 2016, 2017, 2018). The chemical composition of PC, GGBS and FA are shown in Table 1.

Uncrushed 5-20 mm round gravel, supplied by the Fagl Lane quarry (Wales, UK), was used as coarse aggregate in this study. Its water absorption and specific gravity were 1.7% and 2.64, respectively. The fine aggregate, also supplied by the Fagl Lan quarry, was well graded and had water absorption and specific gravity of 2.6% and water absorption of 2.60, respectively. The aggregate grading curves and the overall grading limits from BS882:1992 (BSI, 1992) (now replaced by BS EN 12620:2002+A1:2008 (BSI, 2002a)), are shown in Fig. 1.

Concrete mixes investigated

The concrete mixtures investigated had target 28-day cube compressive strengths of 30 and 50 MPa. The neat PC mixes were PC30 and PC50. Mixes with 30% of the total binder being FA were FA30 and FA50 whilst those with 50% GGBS were GGBS30 and GGBS50. The mixture proportions of the concrete investigated are shown in Table 2 as are the compressive strengths results for standard (20 °C) and adiabatic curing regimes.

Mixing, casting, curing and testing procedures

The concrete mixtures were prepared using horizontal pan mixer with a capacity of 0.1 m³. The cementitious materials and aggregate were firstly introduced to the mixing pan in order to be dry-mixed for approximately one minute. This was then succeeded by adding the water into the mixing pan and the mixing was, subsequently, continued for approximately five minutes. The consistency was evaluated by implementing the slump test in accordance with BS EN 12350-2:2009 (BSI, 2009). Single- and three-gang steel moulds were used to

cast 100 mm and 150 mm size concrete cube specimens. Casting was carried out in two layers with each layer compacted on a vibrating table.

Two different curing methods were applied:

- **Standard curing** – The 100 mm size concrete specimens cast inside single cube moulds were covered with wet hessian and a polythene sheet immediately after casting and left to cure at room temperature conditions (approximately 20 °C) (BSI, 2002b). After 24 hours they were demoulded and placed inside a water bath (20 °C).
- **Adiabatic curing** – The adiabatic temperature rise caused by the cement hydration reaction will occur if heat exchange between fresh concrete and surrounding environment is restricted. To achieve such state, it is required to either provide heavy insulation around the concrete, which will inevitably result in a degree of heat loss, or alternatively, to ensure that the environment surrounding the concrete is at the same, or approximately the same, temperature as the concrete. The latter approach was adopted in this research. A 150 mm concrete cube was cast in a stainless steel box in which 20 mm thick expanded polystyrene was lined for thermal insulation and heavy-duty polythene to prevent any moisture losses. The specimen was subsequently submerged into a programmable computer controlled curing water tank and two Type-T thermocouples were embedded in it through an opening in the top of the box. Two additional Type-T thermocouples were submerged in the water in the tank in order to continuously monitor its temperature. The thermocouples were all connected to a Pico TC08 data logger and a computer which was recording the temperatures and was also programmed to trigger the water heating system once the difference between the water and the concrete sample temperatures was exceeding 1 °C. It may be assumed, taking into consideration the fact that there has been no drop in temperature

once the peak had been reached, that there was only a very low amount of heat lost and thus no adjustment was deemed necessary for the results. However, even small heat losses during hydration may have still affected the peak temperature. The programmable computer control curing tank used for adiabatic tests is shown in Fig. 2 and a schematic diagram of the setup in Fig. 3. In addition, the three gang-moulds, containing 100 mm size concrete “companion” specimens, were wrapped after casting with cling film and tape and submerged in the programmable computer controlled curing tank. This allowed for determination of the compressive strength for the adiabatic curing regime.

The specimens cured under standard curing temperature were tested at 1, 2, 3, 5, 7, 14, 28, 42, 84, 156 and 365 days whilst those cured under adiabatic conditions were tested at 1, 2, 3, 5, 7, 14 and 28 days. For all curing temperatures and testing ages, three specimens were tested in order to derive an average compressive strength.

Results and discussion

The contribution of GGBS and FA to: (a) the strength development under standard (20 °C) curing, (b) the adiabatic temperature rise, and (c) the strength development under adiabatic curing is first examined. Subsequently, the applicability/accuracy of different maturity models for estimating the compressive strength development of concretes with CRMs under adiabatic conditions is investigated.

Strength development at 20 °C

The strength development curves for all the six replicated concrete mixes, *i.e.* PC30 and PC50, GGBS30 and GGBS50, and FA30 and FA50, are shown in Fig. 4(a) whilst the

hyperbolic function suggested by Carino (Carino and Tank, 1992), see Equation 4, which is also the one recommended by ASTM Standard C1074-11 (ASTM, 2011), was used for the regression curves. The regression analysis constants S_u , k and t_0 obtained are shown in Table 3. It appears that FA30 and FA50 concretes had higher 28-day cube compressive strengths than those of the corresponding PC and GGBS mixtures. The contribution of FA to the long-term compressive strength development also becomes apparent in Fig. 4 whilst compressive strength of GGBS mixtures at early ages is again confirmed to be lower compared to the equivalent ones of PC and even FA mixes. Fig. 4(b) shows the compressive strength versus maturity index as calculated by Equation 1.

Adiabatic temperature rise

The adiabatic temperature rise of all the investigated concretes is depicted in Fig. 5(a). The neat PC concretes of 30 and 50 MPa strengths had a temperature rise of 32.5 °C and 48 °C respectively from a placement temperature of nearly 20 °C. 50% GGBS replacement appears to be effective in reducing the adiabatic temperature rise to a considerable extent, i.e. down to 24 °C and 38 °C for grades 30 and 50 MPa, respectively. 30 % FA replacement reduced the temperature of the 50 MPa concrete down to 39 °C but there was no reduction for the 30 MPa concrete. This appears to be abnormal except that FA was not used to replace PC on a weight for weight basis. FA was 30% of the total binder but, because the concretes were designed to have equal 28-day strength, the FA concrete mixes had higher binder contents, see Table 2; 385 kg/m³ for FA50 compared to 330 kg/m³ for PC50 and 275 kg/m³ for FA30 compared to 240 kg/m³ for PC30. Also, to achieve the target strength grades, the w/b ratios of FA concretes were lower than those of comparable PC mixes (Table 2). The total heat emitted per kilogram of binder in the concrete at any time during the adiabatic test can be calculated from the following expression (Ballim and Graham, 2005):

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$$q_b(t) = C_p \times (T_t - T_0) \times \frac{\gamma_c}{b_c} \quad \text{Equation 5}$$

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where: q_b is the heat output per kilogram of binder at time t (kJ/kg),

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C_p is the specific heat capacity of concrete (J/kg·°C), taken as 880 J/kg·°C

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T_t is the temperature of the concrete at time t (°C),

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T_0 is the temperature of the concrete at the beginning of the test (°C),

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γ_c is the density of the concrete (kg/m³).

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b_c is the binder content of the concrete (kg/m³)

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The binder heat output for PC50 and PC30 was 288 kJ/kg and 281 kJ/kg, for FA50

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and FA30 was 210 kJ/kg and 242 kJ/kg and for GGBS50 and GGBS30 was 239 kJ/kg and

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217 kJ/kg, respectively, see Fig. 5(b). Both FA and GGBS reduced the heat output (kJ) per

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kilogram of binder. However, in increasing the binder content to achieve similar 28-day

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strengths to PC, the FA30 had similar temperature rise to that of PC30. The higher strength

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mixtures as expected achieve higher temperatures, despite that the heat output per kilogram

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of binder is slightly reduced at the lower w/b ratios, particularly for the FA concretes. This

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has also been reported by others (Kanavaris, 2017; Turu'allo, 2013; Riding et al. 2012;

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Hatzitheodorou, 2007; Pane and Hansen, 2005; Zhang et al. 2002).

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Effect of adiabatic temperature rise on strength development

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The strength development of all concrete mixes cured under adiabatic conditions is

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shown in Fig. 6. All the concretes benefited from the adiabatic temperature rise and had much

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higher early age strengths than when cured at 20 °C. The most benefit for the GGBS concrete

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appears to be at 2 or even 3 days and this is confirmed by plotting the adiabatic ($S_{\text{adiabatic}}$) to

standard ($S_{20\text{ }^{\circ}\text{C}}$) curing relative strengths as shown in Fig. 7. This is because GGBS reduces not only the temperature rise but also the rate of temperature rise at early ages. The peak temperature of concretes with GGBS is reached after the first day and therefore a more marked improvement in strength is obtained at 2 and 3 days rather than one day.

The improvements in the compressive strengths relative to each mixture's standard 28-day ($20\text{ }^{\circ}\text{C}$) curing strength, i.e. ($S_{\text{adiabatic}}/S_{28\text{-day, }20\text{ }^{\circ}\text{C}}$), see Fig. 8, show that the strengths of FA and GGBS mixes are still lower than the adiabatically cured PC concretes at early ages. This is despite that the strength improvement of FA and GGBS mixes is more significant than the corresponding PC mixes. Even the moderate improvement to PC strengths has maintained their strength above those with GGBS and FA at least for the first day if not for up to 3 days. The “cross-over” effect i.e. high curing temperature results in a greater strength than a low curing temperature at early ages, and conversely results in lower strength at later maturities (McIntosh, 1956), is less apparent for the adiabatically cured specimens than it was for isothermally cured specimens (Soutsos et al. 2017). The PC50 mix shows “cross-over” between three and five days whilst the PC30's only occurs at 28-days. The “cross-over” for GGBS and FA mixes is not apparent within the first 28 days but it is likely that this will occur at later ages, see Fig. 6.

The improvement of strength, in terms of the adiabatic strength ($S_{\text{adiabatic}}$) to the standard curing strength ($S_{20\text{ }^{\circ}\text{C}}$) for the FA mixes seems to be similar to that of PC concretes with 30 MPa strength and remarkably better for the 50 MPa, see Fig. 9. The latter seems to indicate that FA contributes significantly to the strength even at low water to binder ratios whilst at the same time reducing the temperature rise of the concretes.

Concrete strength estimates

The only requirement of Nurse-Saul function in order to calculate the maturity index according to Equation 1 or the equivalent age according to Equation 2, is the temperature history of the concrete. Conversely, in addition to the temperature history, the Arrhenius function also required the “apparent” activation energies, E_a , of concretes under investigation. These were previously determined (Soutsos et al. 2017) according to ASTM C1074-11 (ASTM, 2011) procedure and they were 37.4, 22.5 and 52.8 kJ/mol for PC30, FA30 and GGBS30 and 29.7, 27.3 and 41.6 kJ/mol for PC50, FA50 and GGBS50 respectively. These E_a values are in good agreement with those found in the literature (Soutsos et al. 2013; Poole et al. 2010; Barnett et al. 2007b; Poole et al. 2007; Barnett et al. 2006). Equation 3 was used to calculate the Arrhenius equivalent age t_e at time t . The specified reference temperature, T_s , used was 293 °K (20 °C). T_a (in °K) was the average concrete temperature during time interval Δt , i.e. the recorded adiabatic temperature histories. The calculated equivalent age, t_e , was then substituted for t in Equation 4 with regression constants S_u , k and t_0 , as previously determined for the strength results obtained for the concrete cured at the reference temperature (20 °C), see Table 2.

The adiabatic temperature histories, Fig. 6, were converted, using the strength-time and Arrhenius equations, into estimated strength development curves and these are shown in Fig. 10 and Fig. 11. The Nurse-Saul function underestimated the strength development, especially at early ages, for all but 50 MPa strength concretes. This is in agreement with previous findings for isothermally cured “equivalent” mortars at 50 °C which are also shown on Fig. 12 and Fig. 13 for comparison (Soutsos et al. 2017). However, the overestimation of strength at later ages is delayed for adiabatic curing regime. The overestimation of strengths is due to the incapability of the Nurse-Saul function to account for the detrimental effect high temperatures at early ages have on later age strength (Soutsos et al. 2017). As mentioned

earlier, in the adiabatic curing regime, the temperature rise depends on the heat evolution from the cementitious binder. The temperature increase is delayed by several hours and notable rises only occur even more hours later as a result of the dormant period (Shi et al. 2006). As the hydration reaction is required to have evolved significantly prior to any high temperatures occurring, the detrimental effect on long-term strengths is considerably reduced. It is also for this reason that it is suggested that curing cycles, e.g. for precast concrete elements, should have a “delay period” before the “temperature rise period” (Neville and Brooks, 2011).

The strength estimates from the Arrhenius function for the adiabatically cured concretes show a significant improvement to the estimates for the isothermally cured concretes (Soutsos et al. 2017), see Fig. 12 and Fig. 13. The improvement is with regards to the over-estimation of strengths, at even early ages, which was suspected to be due to the detrimental effect of high curing temperatures starting from early age for isothermally cured mortars (Soutsos et al. 2017). The only concrete that still showed detrimental effect from high early age curing temperature was PC50 which had a significant temperature rise of 37 °C within 12 hours after casting. The Arrhenius function overestimates the early age strengths for this concrete but to a lesser extent than it did for the 50 °C isothermally cured specimens.

Conclusions

The strength development of isothermally (20 °C) and adiabatically cured concretes was determined. It was found that:

- GGBS was efficient in reducing considerably the adiabatic peak temperature rise.
- FA was only efficient in reducing considerably the adiabatic peak temperature rise for the high, 50 MPa, compressive strength which had a lower water-binder ratio.

• Significant increases in early age strength resulted from the adiabatic curing regime despite that considerable temperature rises did not occur until after 12 hours and peak temperatures only after 24 hours.

• The “delay period” before the “temperature rise period” of the adiabatic curing regime was sufficient to reduce or delay the “cross-over” effect to beyond 28-days for all mixes other than PC50.

Maturity functions were used to estimate the strengths for the adiabatically cured concretes. It was found that:

• The Nurse-Saul function underestimated the improvements in the early age strengths resulting from the higher “curing” temperatures of the adiabatic curing regime. It is believed that this occurred because it assumes that the concrete strength gain rate varies linearly with temperature and is the same for all binders.

• The Arrhenius based function was found to be more accurate and this is because it allows for an exponential strength gain rate with temperature relationship.

• The Arrhenius based function strength estimates were significantly better for the adiabatically cured concretes than for the 50 °C isothermally cured ones. The latter are believed to have been affected by the detrimental effect of high curing temperatures starting from early age.

Ongoing work is aiming to determine modifications to the currently available maturity functions or develop improved ones for better estimates of both early age and long-term strength development with and without cement replacement materials.

Acknowledgements

The majority of the experimental work described here was carried out by Dr A. Hatzitheodorou at the University of Liverpool as part of his PhD research. The authors are grateful to the School of Engineering, the University of Liverpool for the facilities provided and to the Engineering and Physical Sciences Research Council, UK (GR/R83880/01), for the financial support received for the equipment. The authors would like to thank Dr L.K.A. Sear at United Kingdom Quality Ash Association (UKQAA) for the extensive advice received during the project.

Notations

The following symbols are used in this paper:

E_a = “apparent” activation energy (J/mol);

k = the rate constant (1/day);

M = Nurse-Saul maturity ($^{\circ}\text{C}\cdot\text{days}$);

R = universal gas constant ($\text{J}/^{\circ}\text{K}\cdot\text{mol}$);

S = compressive strength (MPa);

S_u = ultimate compressive strength (MPa);

T = average temperature ($^{\circ}\text{K}$ or $^{\circ}\text{C}$);

T_0 = datum temperature ($^{\circ}\text{C}$);

T_r = specified reference temperature ($^{\circ}\text{K}$ or $^{\circ}\text{C}$);

t_0 = age at which compressive strength development is assumed to begin (days);

t_e = equivalent age (days);

β = age conversion factor;

Δt = time interval (days).

References

- ASTM (2011). "Standard practice for estimating concrete strength by the maturity method." *ASTM C1074-11*, West Conshohocken, Pennsylvania, USA, ASTM International.
- Ballim, Y., and Graham, P.C. (2005). "A maturity approach to the heat evolution of concrete." *Magazine of Concrete Research*, **55(3)**, pp. 249–256.
- Barnett, S.J., Soutsos, M.N., Millard, S.G., and Bungey, J.H. (2006). "Strength development of mortars containing ground granulated blast-furnace slag: Effect of curing temperature and determination of apparent activation energies." *Cement and Concrete Research*, **36(3)** pp. 434–440.
- Barnett, S.J., Soutsos, M.N., Bungey, J.H., and Millard, S.G. (2007a). "Fast-Track Construction with Slag Cement Concrete: Adiabatic Strength Development and Strength Prediction." *ACI Materials Journal*, **104(4)**, pp. 388–396.
- Barnett, S.J., Soutsos, M.N., Millard, S.G., and Bungey, J.H. (2007b). "Temperature Rise and Strength Development in Laboratory-Cast Structural Elements Containing Slag." *American Concrete Institute SP-242: Ninth CANMET/ACI International Conference on Fly Ash, Silica Fume, Slag and Natural Pozzolans in Concrete* (Malhotra VM

441 (ed)), Warsaw, Poland, ISBN 978-0-87031-241-0, pp. 37–50.

442 Brooks, A.G., Schindler, A.K., and Barnes, R.W. (2007). “Maturity method evaluated for
443 various cementitious materials.” *Journal of Materials on Civil Engineering*, **19(12)**,
444 pp. 1017–1025.

445 BSI (British Standards Institution). (1992). “Specification for aggregates from natural sources
446 for concrete.” *BS 882:1992*, London, UK.

447 BSI (British Standards Institution). (2016). “Concrete – Complementary British standard to
448 BS EN 206 – Part 1: Method of specifying and guidance for the specifier.” *BS 8500-
449 1:2015+A1:2016*, London, UK.

450 BSI (British Standards Institution). (2011). “Cement. Composition, specifications and
451 conformity criteria for common cements.” *BS EN 197-1:2011*, London, UK.

452 BSI (British Standards Institution). (2009). “Testing fresh concrete. Slump-test.” *BS EN
453 12350-2:2009*, London, UK.

454 BSI (British Standards Institution). (2002a). “Aggregates for concrete.” *BS EN
455 12620:2002+A1:2008*, London, UK.

456 BSI (British Standards Institution). (2002b). “Testing hardened concrete. Compressive
457 strength of test specimens.” *BS EN 12390-3:2002*, London, UK.

458 BSI (British Standards Institution). (2005). “Methods of testing cement. Determination of
459 strength.” *BS EN 196-1-2005*, London, UK.

460 BSI (British Standards Institution). (2012). “Fly ash for concrete. Definition, specifications
461 and conformity criteria.” *BS EN 450-1:2012*, London, UK.

462 BSI (British Standards Institution). (2016). “Design and use of insert of lifting and handling
463 of precast concrete elements.” *PD CEN/TR 15728:2016*, London, UK.

464 BSI (British Standards Institution). (2006) “Ground granulated blast furnace slag for use in
465 concrete, mortar and grout. Definitions, specifications and conformity criteria.” *BS*

466 *EN 15167-1:2006*, London, UK.

467 Carino, N.J., and Tank, R.C., (1992). “Maturity Functions for Concretes Made with Various
468 Cements and Admixtures.” *ACI Material Journal*, **89(2)**, pp. 188–196.

469 Carino, N.J. (2004). “The Maturity Method, In: Handbook on Nondestructive Testing of
470 Concrete.” (Malhotra VM and Carino NJ (eds)), 2nd ed. *CRC Press*, pp. 5.1–5.47.

471 Feld, J., and Carper, K. (1997). “Construction Failure.” *John Wiley and Sons*, New York.

472 Freiesleben, H.P., and Pedersen, E.J. (1977). “Maturity computer for controlled curing and
473 hardening of concrete.” *Nord. Betong*, **1(19)**.

474 Freiesleben, H.P., and Pedersen, E.J. (1985). “Curing of concrete structures.” *CEB*
475 *Information Bulletin*, **166**.

476 Galobardes, I., Cavalaro, S., Goodier, C.I., Austin, S., and Rueda, Á. (2015). “Maturity
477 method to predict the evolution of the properties of sprayed concrete.” *Construction*
478 *and Building Materials*, **79**, pp. 357–369.

479 Hatzitheodorou, A. (2007). “Insitu Strength Development of Concretes with Cement
480 replacement materials.” *PhD thesis*, University of Liverpool, UK.

481 Kanavaris, F. (2017). “Early age behaviour and cracking risk of concretes containing GGBS.”
482 *PhD Thesis*, Queen’s University Belfast, UK.

483 Kim, J.-K., Moon, Y.-H., and Eo, S.-H. (1998). “Compressive strength development of
484 concrete with different curing time and temperature.” *Cement and Concrete Research*,
485 **28(12)**, pp. 1761-1773.

486 Lew, H., Fattel, S., Shaver, J., Reinhold, T., and Hunt, B. (1979). “Investigation of
487 construction failure of reinforced concrete cooling tower at Willow island.” *West*
488 *Virginia Final Report (No. PB-80-192883)*, National Engineering Lab (NBS),
489 Washington.

490 Lothenbach, B., Winnefeld, F., Alder, C., Wieland, E., and Lunk, P. (2007). “Effect of

temperature on the pore solution, microstructure and hydration products of Portland cement pastes.” *Cement and Concrete Research*, **37(4)**, pp. 483-491.

McIntosh, J.D. (1956). “The effects of low-temperature curing on the compressive strength of concrete.” In *Proceedings of RILEM symposium on winter concreting*, Session BII, Copenhagen: Danish Institute for Building Research.

Neville, A.A., and Brooks, J.J. (2010). “Concrete Technology.” *Pearson Education*, 2nd Edition, p. 188.

Pane, I., and Hansen, W. (2005). “Investigation of blended cement hydration by isothermal calorimetry and thermal analysis.” *Cement and Concrete Research*, **35**, pp. 1155–1164.

Poole, J.L., Riding, K.A., Folliard, K.J., Juenger, M.C.G., and Schindler, A.K. (2007). “Methods for calculating apparent activation energy for Portland cement.” *ACI Materials Journal*, **104(1)**, pp. 303–311.

Poole, J.L., Riding, K., Juenger, M.C.G., Folliard, K.J., and Schindler, A.K. (2010). “Effects of cement replacement materials on apparent activation energy.” *Journal of ASTM International*, **7(9)**, 2010.

Rastrup, E. (1954). “Heat of hydration in concrete.” *Magazine of Concrete Research*, **6(17)**, pp. 79–92.

Reddy, J., and Soutsos, M. (2016). “Thermal activation of low carbon precast concrete.” In: Jones, M.R., Newlands, M.D., Halliday, J.E., Csetenyi, L.J., Zheng, L., McCarthy, M.J. and Dyer, T.D., editors, *Proceedings of the 9th International Concrete Conference*, University of Dundee, Scotland, UK, July, pp. 158–171.

Riding, K.A., Poole, J.L., Folliard, K.J. Juenger, M.C.G., and Schindler, A.K. (2012). “Modeling hydration of cementitious systems.” *ACI Materials Journal*, **109(2)**, pp. 225–234.

516 Sajedi, F., and Razak, H.A. (2011). "Effects of curing regimes and cement fineness on the
517 compressive strength of ordinary Portland cement mortars." *Construction and*
518 *Building Materials*, **25**(4), pp. 2036-2045.

519 Saul, A.G.A., (1951). "Principles underlying the steam curing of concrete at atmospheric
520 pressure." *Magazine of Concrete Research*, **2**(6), pp. 127–140.

521 Shi, C., Krivenko, P., and Roy, D. (2006). "Alkali-Activated Cements and Concretes." *Taylor*
522 *& Francis*, p. 65.

523 Sofi, M., Mendis, P.A., and Baweja, D. (2012). "Estimating early-age in situ strength
524 development of concrete slabs." *Construction and Building Materials*, **29**, pp. 659–
525 666.

526 Soutsos, M.N., Turu'allo, G., Owens, K., Kwasny, J., Barnett, S.J., and Basheer, P.A.M.
527 (2013). "Maturity Testing of Lightweight Self-Compacting and Vibrated Concretes."
528 *Construction and Building Materials*, **47**, pp. 118–125.

529 Soutsos, M.N., Hatzitheodorou, A., Kwasny, J., and Kanavaris, F. (2016). "Effect of in situ
530 temperature on the early age strength development of concretes with cement
531 replacement materials." *Construction and Building Materials* **103**, pp. 105–116.

532 Soutsos, M.N., Hatzitheodorou, A., Kanavaris, F., and Kwasny, J. (2017). "Effect of
533 temperature on the strength development of mortar mixes with GGBS and fly ash."
534 *Magazine of Concrete Research*, **69**(15), pp. 787–801.

535 Soutsos, M., Kanavaris, F. and Hatzitheodorou, A. (2018). "Critical analysis of strength
536 estimates from maturity functions", *Case Studies in Construction Materials*, 9, 2018,
537 e00183.

538 The Concrete Society (2004). "In situ concrete strength. An investigation into the relationship
539 between core strength and standard cube strength." Concrete Society Project Report
540 No. 3, The Concrete Society, Crowthorne, UK, 55 pp.

Turu'allo, G. (2013). "Early age strength development of GGBS concrete cured under different temperatures." *PhD thesis*, University of Liverpool, UK.

Yikici, T.A., and Chen, H. (2015). "Use of maturity method to estimate compressive strength of mass concrete." *Construction and Building Materials*, **95**, pp. 802–812.

Zhang, Y., Sun, W. and Liu, S. (2002). "Study on the hydration heat of binder paste in high-performance concrete." *Cement and Concrete Research*, **32**, pp. 1483–1488.

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572

573 **Tables**574 **Table 1.** Chemical composition of PC, GGBS and FA

Chemical composition (% by weight)			
Chemical constituent	PC	GGBS	FA
SiO ₂	20.11	35.35	48
Al ₂ O ₃	5.16	14	27
Fe ₂ O ₃	3.14	0.36	9
CaO	65.49	41.41	3.3
MgO	0.8	7.45	2
SO ₃	3.22	0.1	0.6
K ₂ O	0.59	-	3.8
Na ₂ O	0.13	-	1.2
CaCO ₃	4.47	-	-
Equiv. Alks Na ₂ Oe	0.52	-	-
Free Lime	1.79	-	-
Chloride	71 ppm	-	-
LOI	2.8	0.31	4.9

575

576

577 **Table 2.** Mix proportions of concrete mixes investigated as well as their compressive strength
 578 for 20 °C and adiabatic curing regimes

Material	Mix ID											
	PC30	GGBS30	FA30	PC50	GGBS50	FA50						
Cement [kg/m³]	240	115	193	345	165	270						
GGBS [kg/m³]	-	115	-	-	165	-						
FA [kg/m³]	-	-	82	-	-	115						
Gravel [kg/m³]	1102	1187	1319	1205	1151	1250						
Sand [kg/m³]	799	721	560	615	683	533						
Free water [kg/m³]	158	150	144	160	165	135						
Total water [kg/m³]	198	190	181	197	203	171						
Free w/b	0.66	0.65	0.52	0.46	0.50	0.35						
Total w/b	0.83	0.82	0.66	0.57	0.61	0.44						
Slump [mm]	150	120	120	135	120	100						
Testing age [days]	Compressive strength [MPa] (20 °C <i>Adiabatic</i>)											
	20 °C	Ad.	20 °C	Ad.	20 °C	Ad.	20 °C	Ad.	20 °C	Ad.	20 °C	Ad.
1	7	13	2	3	7	12	21	31	5	12	13	24
2	13	22	4	9	13	20	30	35	10	28	22	38
3	18	25	7	14	17	25	35	37	15	37	28	46
5	20	26	10	21	21	32	41	39	21	43	34	53
7	23	29	13	25	24	34	43	39	26	42	38	56
14	28	29	19	29	28	39	47	41	34	45	43	56
28	31	31	25	31	35	39	50	49	42	47	51	57
42	32	-	28	-	37	-	53	-	43	-	58	-
84	33	-	31	-	46	-	55	-	51	-	59	-
156	33	-	34	-	47	-	59	-	-	-	66	-
365	32	-	37	-	47	-	56	-	-	-	67	-

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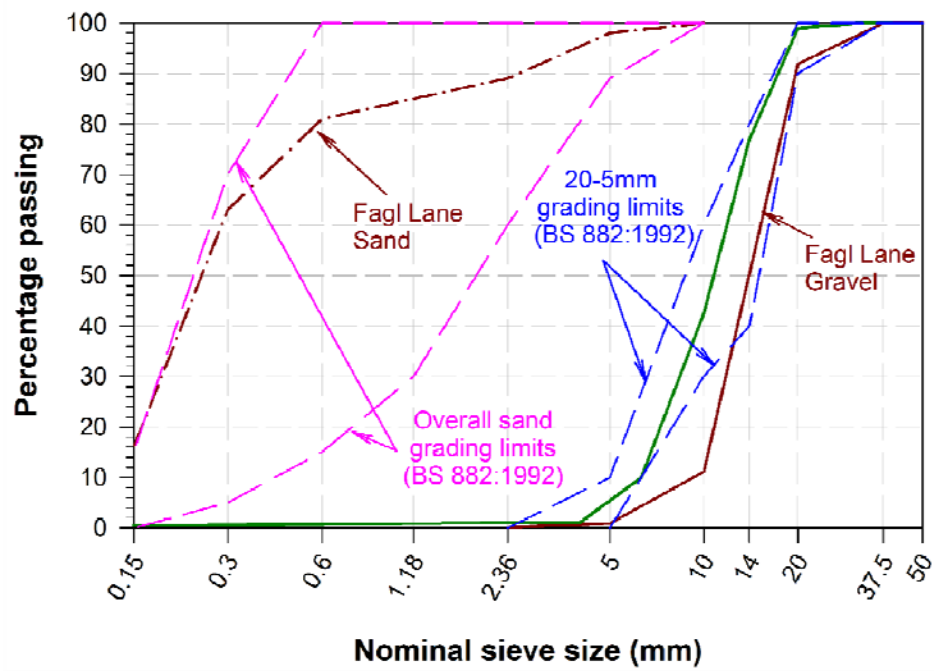
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581 **Table 3.** Regression parameters (obtained from Equation 4) for 20 °C strength development

<i>Mix ID</i>	<i>PC30</i>	<i>GGBS30</i>	<i>FA30</i>	<i>PC50</i>	<i>GGBS50</i>	<i>FA50</i>
S _u (MPa)	33.36	36.91	46.35	55.49	55.70	63.93
k (1/day)	0.37	0.08	0.15	0.56	0.11	0.22
t ₀ (days)	2.45E-01	1.99E-01	7.50E-09	2.49E-09	1.30E-09	6.33E-09

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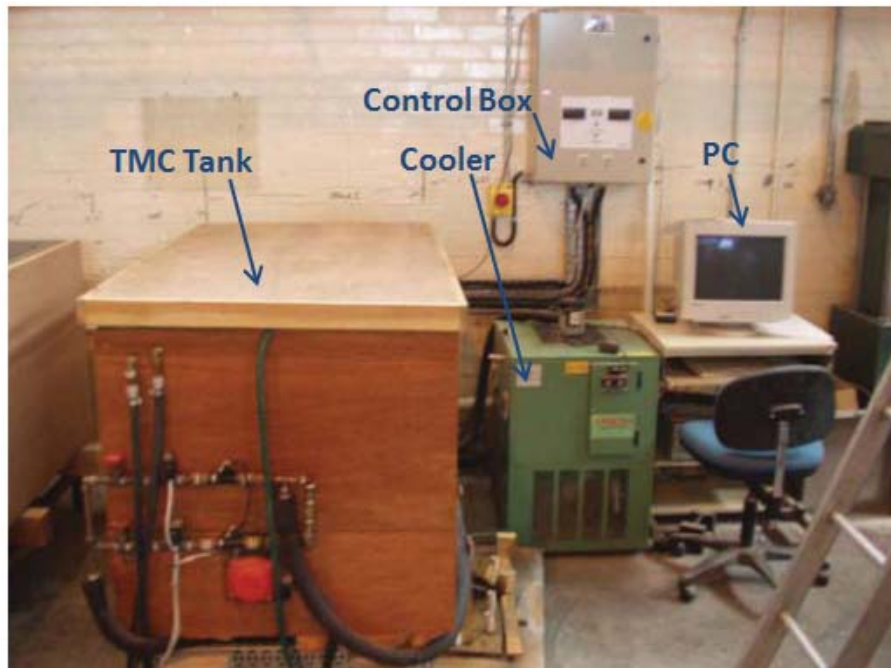
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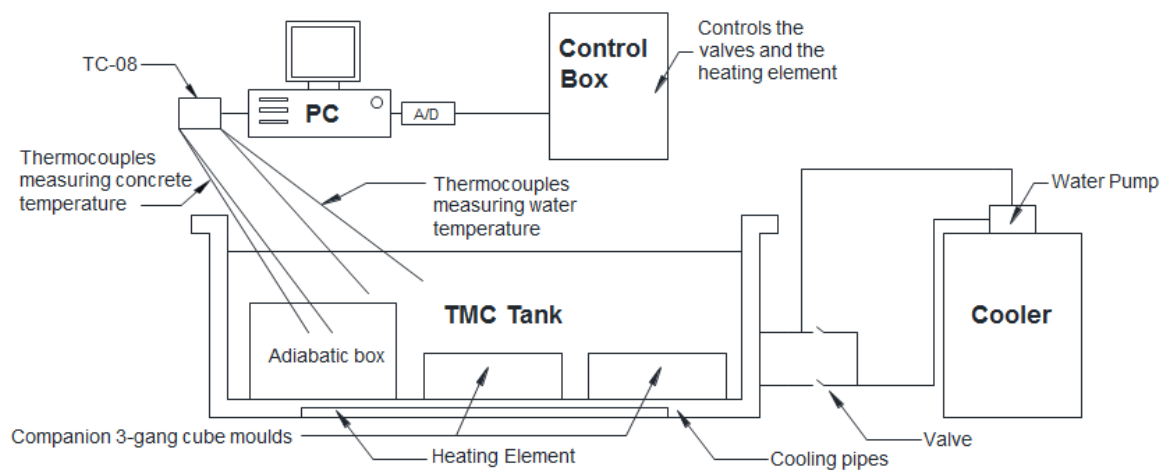
585 **Fig. 1.** Sieve analysis of coarse and fine aggregate

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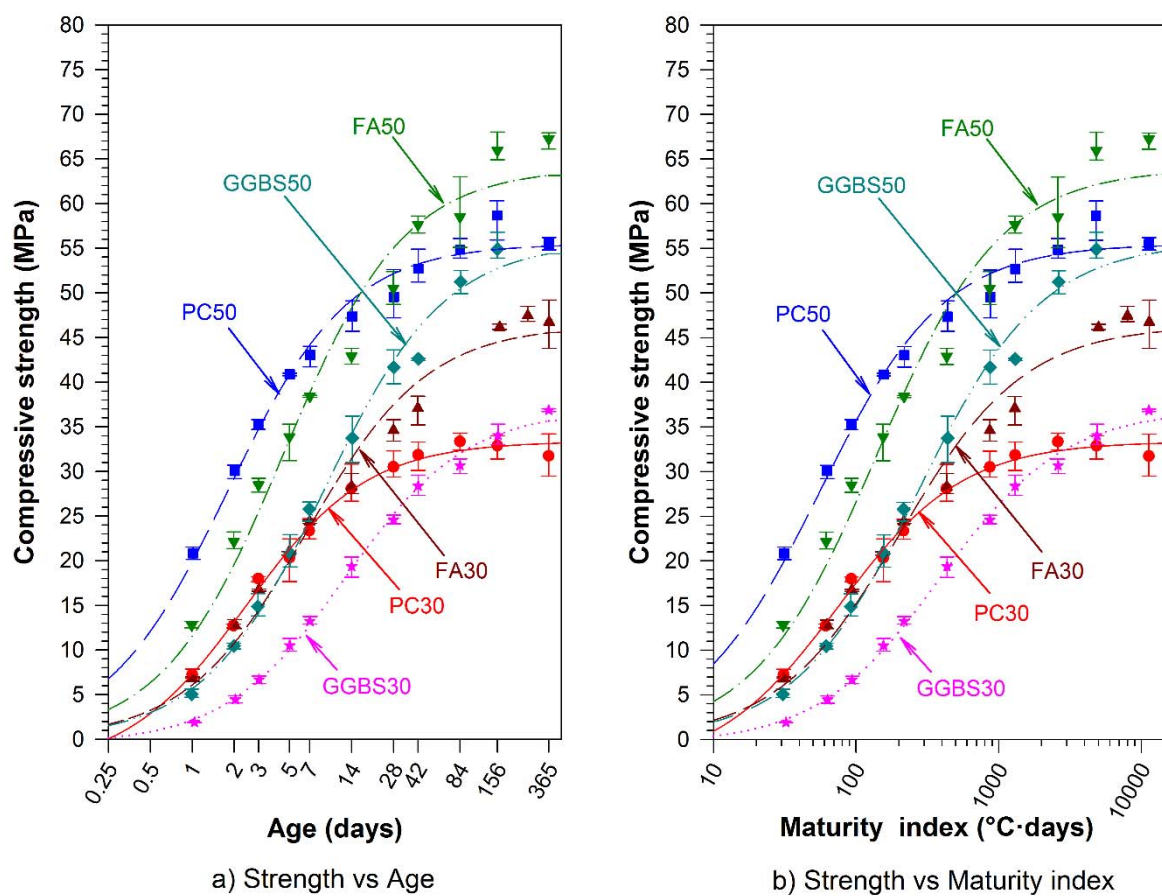


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588 **Fig. 2.** The computer controlled temperature matched curing (TMC) tank for adiabatic tests



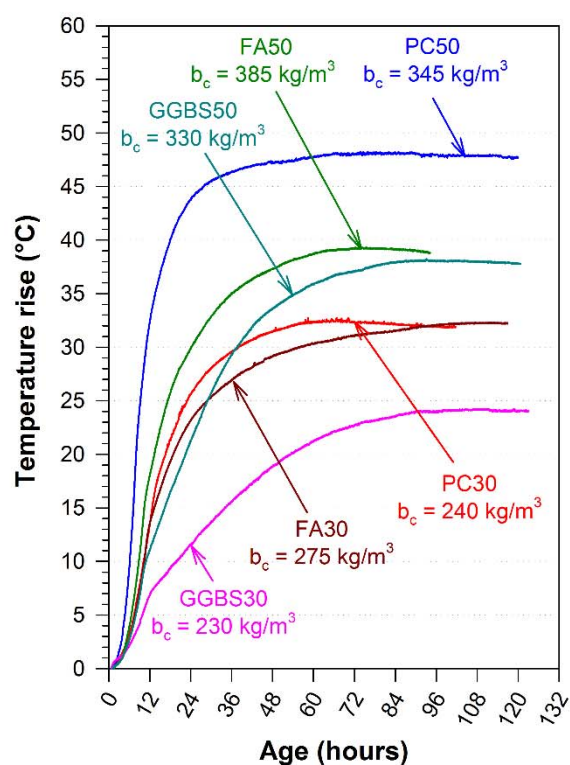
591 **Fig. 3.** Schematic diagram of the computer controlled TMC tank setup for the adiabatic tests



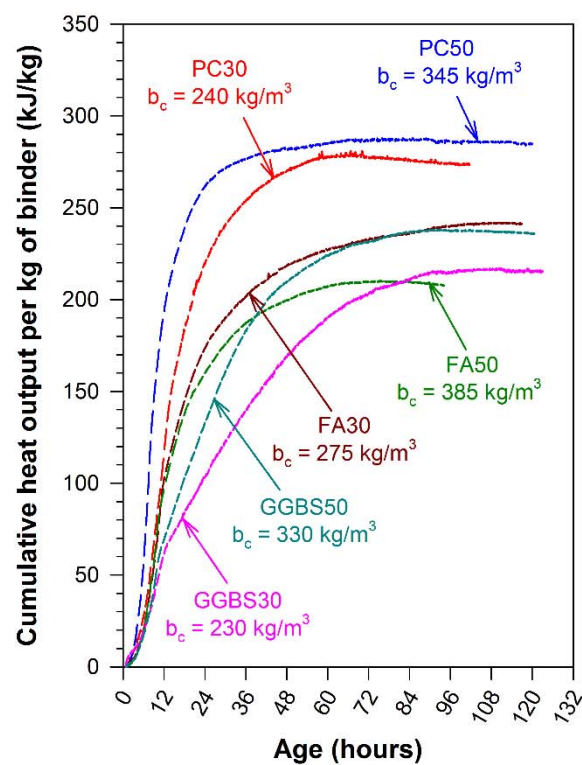
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594 **Fig. 4.** Strength development regression analysis plots of laboratory replicated DTI concrete

595 mixes (standard 20 °C curing)



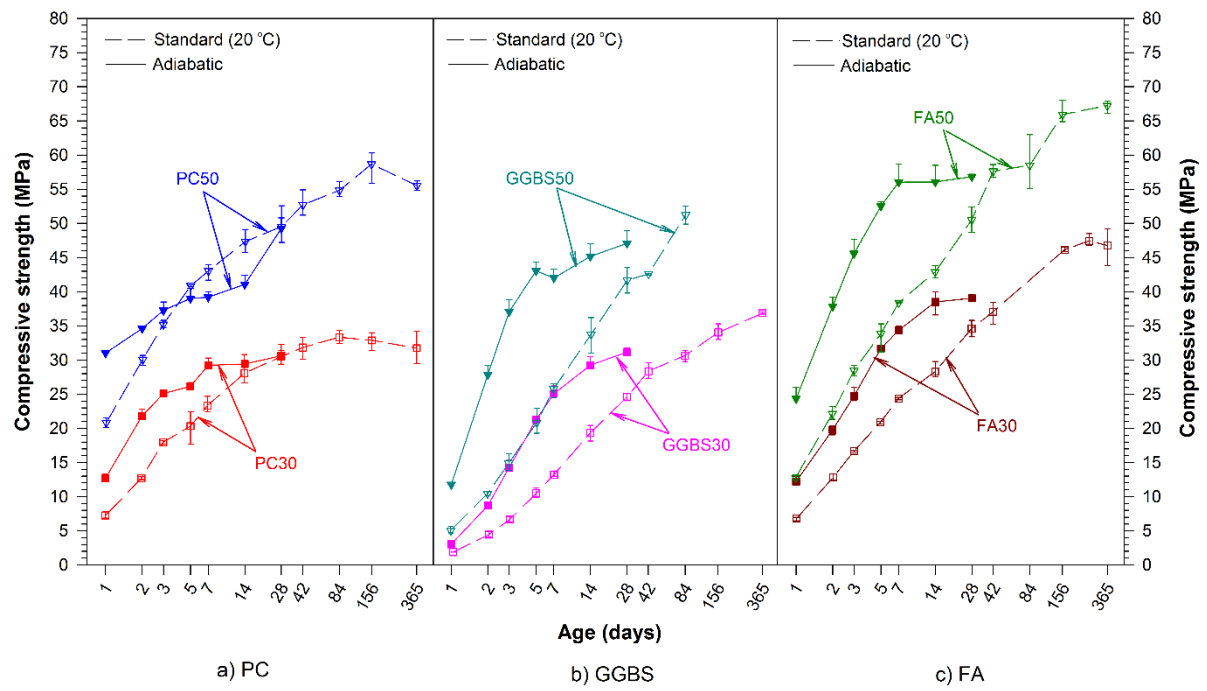
a) Adiabatic temperature rise



b) Cumulative heat output

Fig. 5. a) Adiabatic temperature rise of investigated mixes and b) Total cumulative heat output of binder

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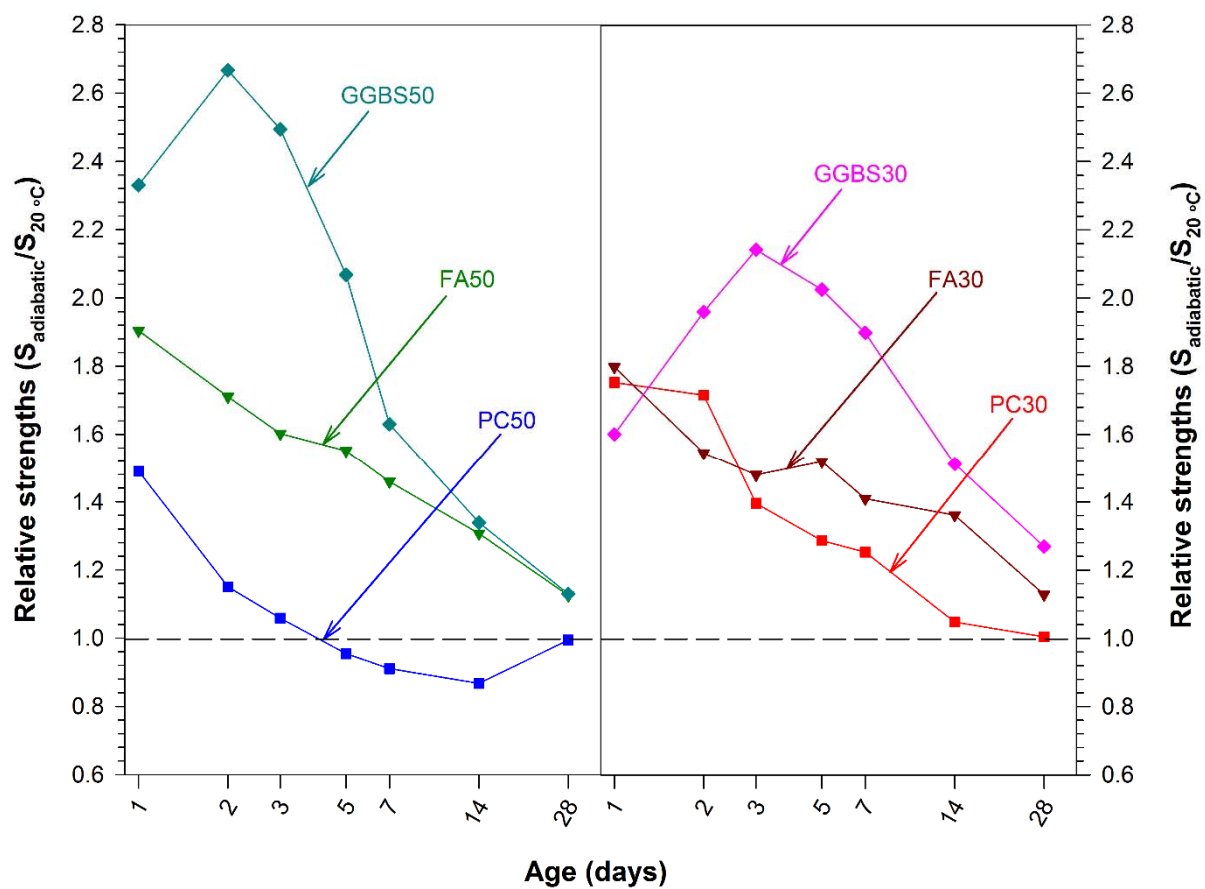
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Fig. 6. Strength development of standard (20 °C) ($S_{20\text{ }^{\circ}\text{C}}$) and adiabatically ($S_{\text{adiabatic}}$) cured concretes

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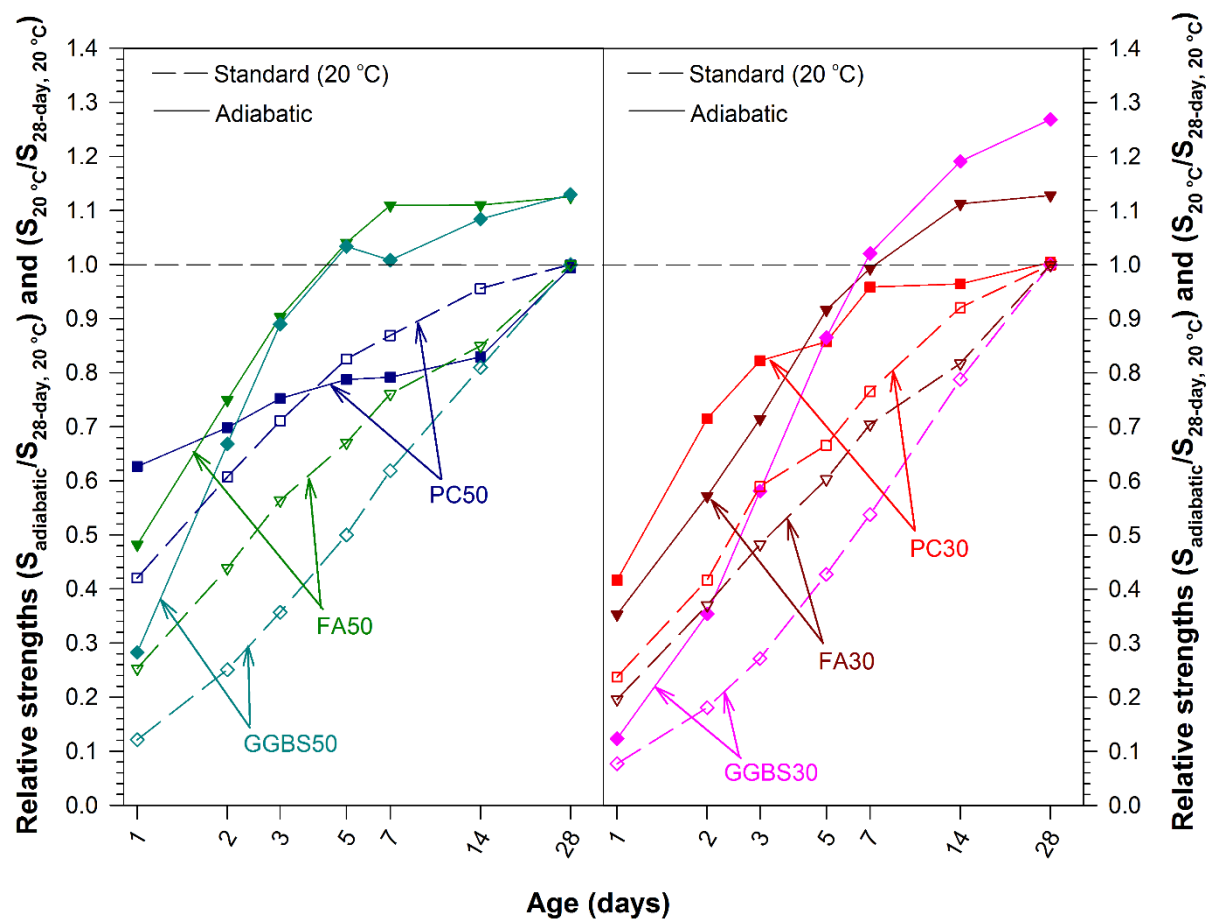
a) 50 MPa concretes

b) 30 MPa concretes

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606 **Fig. 7.** Relative strengths, i.e. ($S_{adiabatic}/S_{20\text{ }^{\circ}\text{C}}$)

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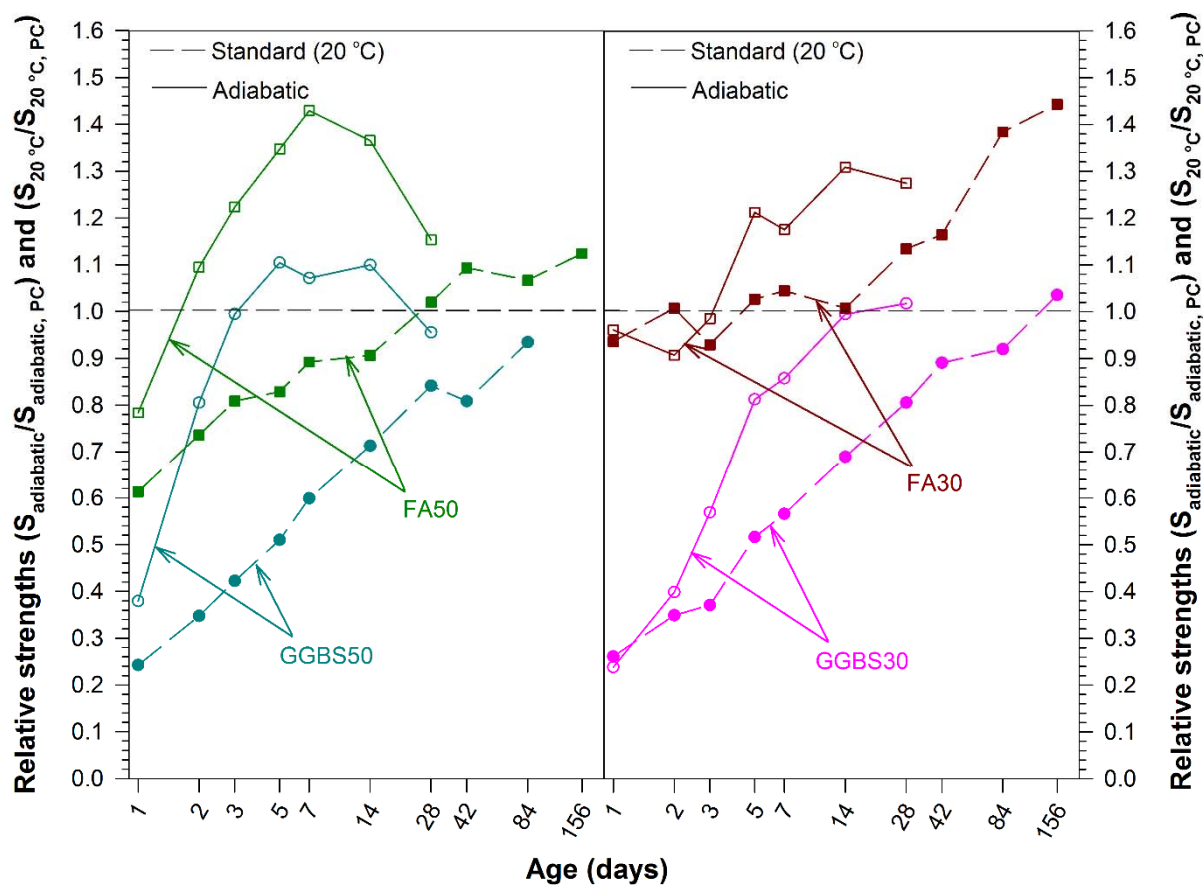
a) 50 MPa concretes

b) 30 MPa concretes

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Fig. 8. Relative strengths, i.e. ($S_{20\text{ }^{\circ}\text{C}}/S_{28\text{-day, }20\text{ }^{\circ}\text{C}}$) and ($S_{\text{adiabatic}}/S_{28\text{-day, }20\text{ }^{\circ}\text{C}}$)

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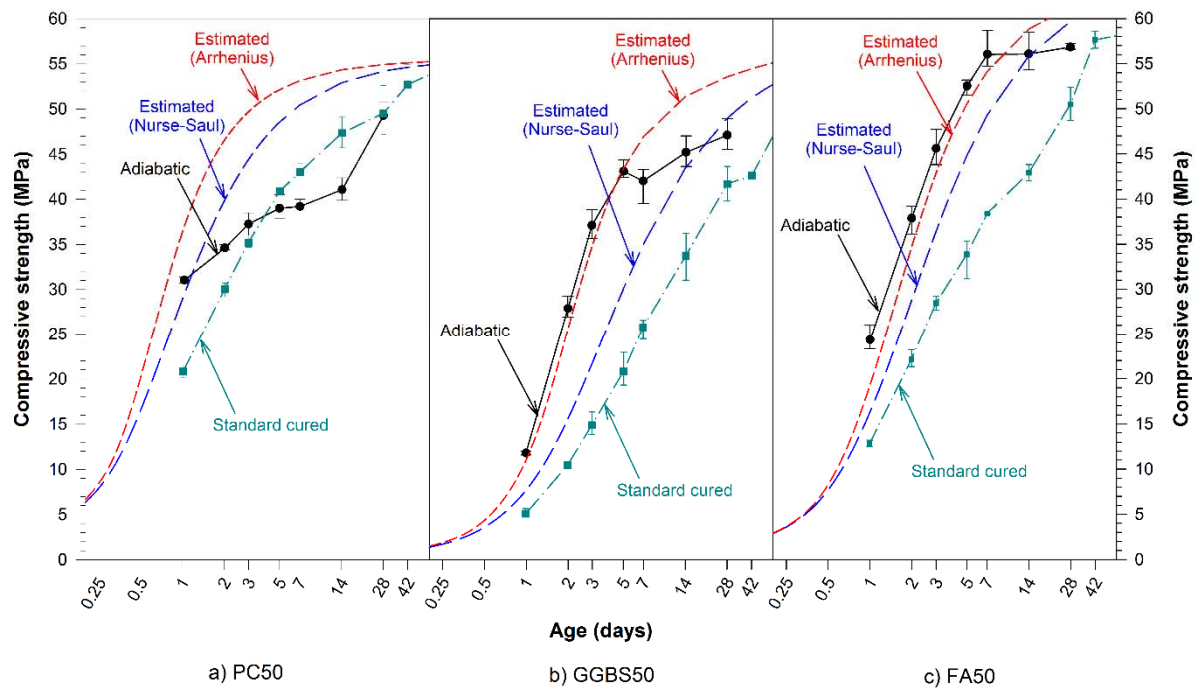
a) 50 MPa concretes

b) 30 MPa concretes

612

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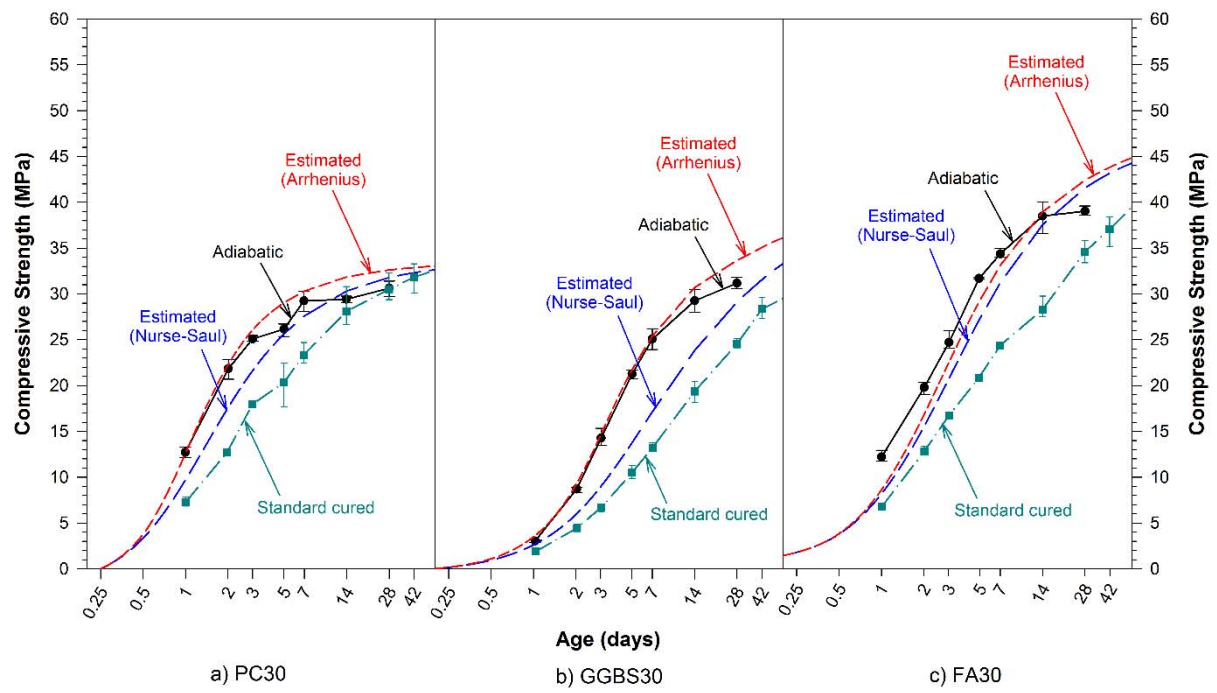
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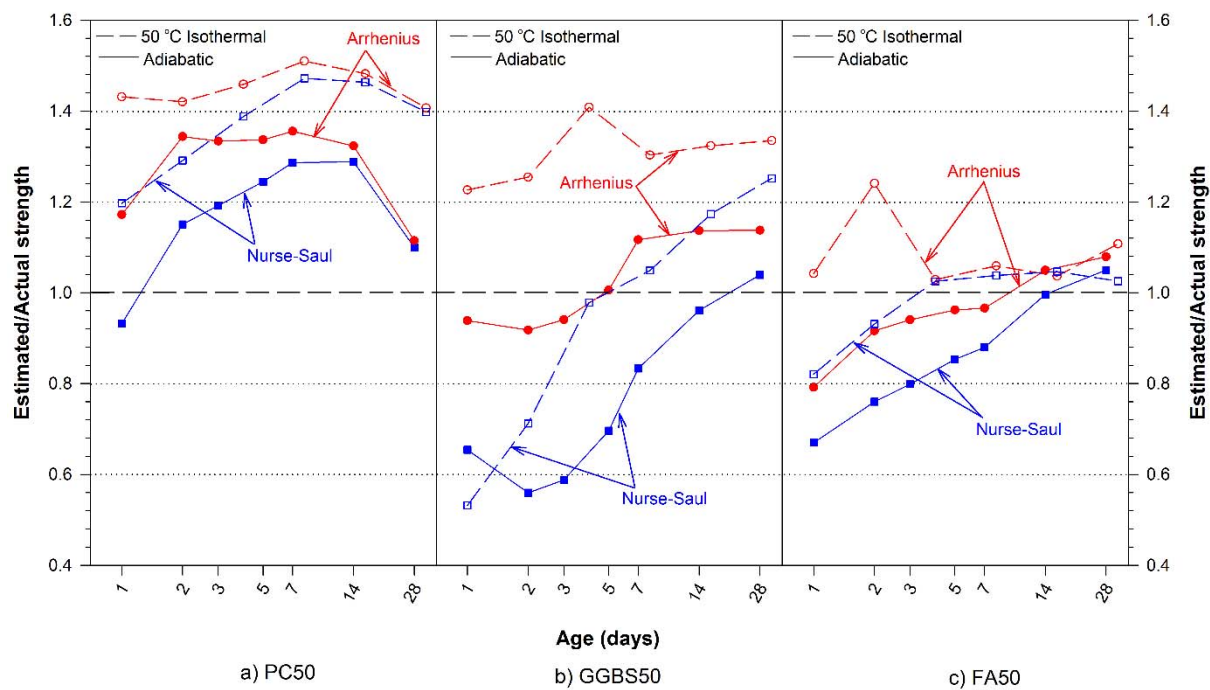
615 **Fig. 10.** Adiabatic strength estimates for 50 MPa strength concretes

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618 **Fig. 11.** Adiabatic strength estimates for 30 MPa strength concretes

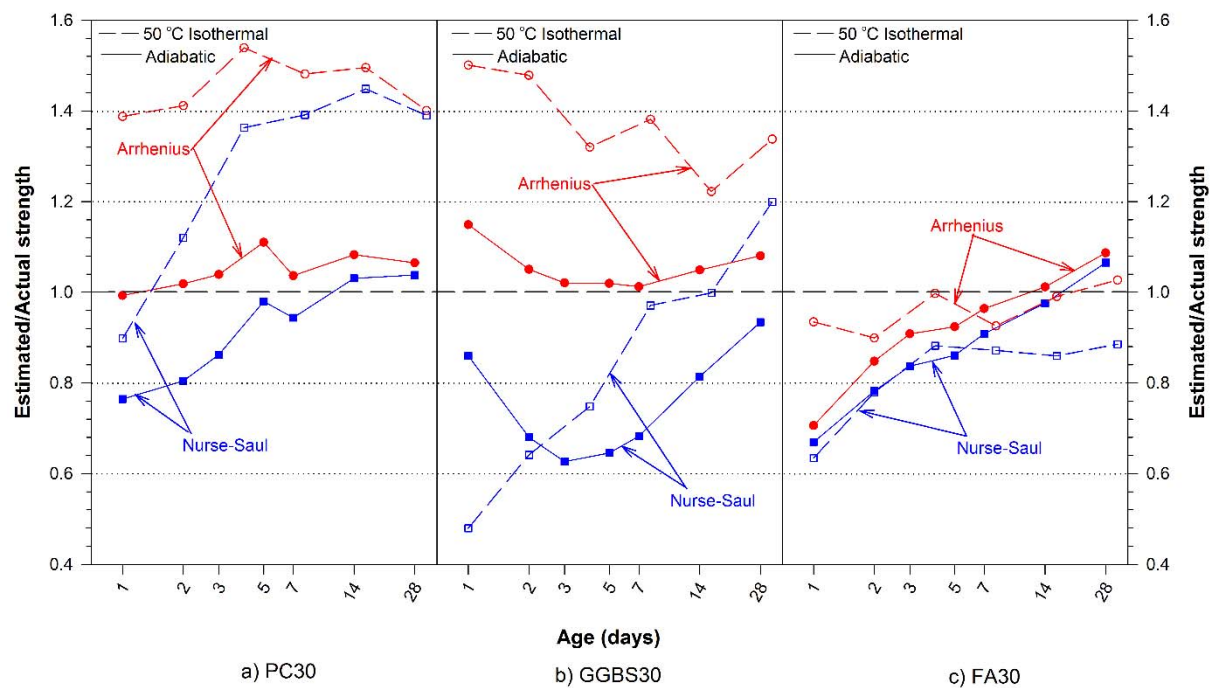


620

621 **Fig. 12.** Ratio of estimated/actual strength for 50 MPa strength adiabatically cured concretes

622 and also for their 50 °C isothermally cured “equivalent” mortars

623



624

625 **Fig. 13.** Ratio of estimated/actual strength for 30 MPa strength adiabatically cured concretes

626 and also for their 50 °C isothermally cured “equivalent” mortars